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IS 14344 (1996): Design and construction of diaphragms for under seepage control - Code of practice [WRD 8: Foundation and Substructures]



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रिसाव नियंत्रण के लिए डायाफ्राम वाल की
डिजाइन तथा निर्माण — रीति संहिता

Indian Standard

DESIGN AND CONSTRUCTION OF
DIAPHRAGMS FOR UNDER-SEEPAGE
CONTROL — CODE OF PRACTICE

ICS 93.160

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BUREAU OF INDIAN STANDARDS
MANAK BHAVAN, 9 BAHADUR SHAH ZAFAR MARG
NEW DELHI 110002

FOREWORD

This Indian Standard was adopted by the Bureau of Indian Standards, after the draft finalized by the Foundation and Substructures Sectional Committee had been approved by the River Valley Division Council.

Construction of conventional rolled earthfill cut-off benches can be a prohibitively expensive operation depending on the required depth and the existing natural ground water condition. The recent trend has been towards application of diaphragm walls in the control of under-seepage.

Diaphragm walls can be constructed by a variety of methods that do not require foundation dewatering and greatly reduce the amount of excavation from that required for a rolled earthen cut-off.

In theory, all the major types of diaphragm walls can be designed to provide a positive seepage cut-off. However, there are certain uncertainties in the design and construction of all types of diaphragm walls. Each application of a diaphragm wall is unique and depends largely on the intended function as on the foundation conditions, hydraulic gradients and economic considerations. In this standard the design and construction aspects of diaphragm walls have been addressed to the situation where the diaphragm wall is used as a measure for under-seepage control.

For the purpose of deciding whether a particular requirement of this standard is complied with, the final value, observed or calculated, expressing the result of a test or analysis, shall be rounded off in accordance with IS 2 : 1960 'Rules for rounding off numerical values (*revised*)'. The number of significant places retained in the rounded off value should be the same as that of the specified value in this standard.

Indian Standard

DESIGN AND CONSTRUCTION OF DIAPHRAGMS FOR UNDER-SEEPAGE CONTROL — CODE OF PRACTICE

1 SCOPE

1.1 This standard covers the design and construction of diaphragm walls to be used as positive impervious cut-offs for dams, weirs and barrages as an under-seepage control measure.

1.2 Diaphragm walls required for other structures are not covered in this standard

2 REFERENCES

2.1 The Indian Standards given in Annex A are necessary adjuncts to this standard.

2.2 For the definitions of various terms used in this standard, reference may be made to relevant parts of IS 4410.

3 GENERAL CONSIDERATIONS FOR DIAPHRAGM WALL AS AN UNDER-SEEPAGE CONTROL MEASURE

3.1 Under-seepage control measures for dams, weirs and barrages have two independent functions:

- a) To reduce the loss of water by seepage, through sub-surface, to an amount compatible with the purpose of the project; and
- b) To eliminate the possibility of structural failure by piping.

3.2 The quantity of seepage can be reduced appreciably by intercepting the pervious strata by means of diaphragm wall. Bottom of the diaphragm wall should be embedded in continuous impervious stratum. Top of the diaphragm wall should be connected with an impervious member of the structure like the impervious core of embankment.

3.3 Chances of failure of dam by piping increases rapidly with increasing value of hydraulic gradient. Careful selection of the type of diaphragm wall can reduce the probability of piping failure to a minimum.

3.4 Selection of under-seepage control measures is dependent on the following factors:

- a) Heterogeneity of sub-surface strata,
- b) Uncertainties in foundation characteristics,

- c) The economic value of the water stored,
- d) The risk element as influenced by the height of dam,
- e) Reservoir volume, and
- f) Potential damage to downstream areas.

4 NECESSITY OF DIAPHRAGM WALL

4.1 Where construction of positive cut-off trench by conventional open excavation is not feasible on account of the situations described below, construction of diaphragm wall as an under-seepage control measure can solve the problem:

- a) Excessive depth of the impervious stratum overlain by pervious or heterogeneous strata;
- b) Construction difficulties, like heavy dewatering;
- c) Instability of side slopes of excavations;
- d) Paucity of construction materials; and
- e) Time constraints, etc.

4.2 It is, however, imperative to study the cost factor, water loss that can be tolerated, risk involved in case of failure of structure, ease in construction, time factor, and any other inherent issues for the project, prior to selecting the under-seepage control measure.

5 TYPES OF DIAPHRAGM WALL

5.1 Depending on the use of construction materials there are the following types of diaphragm walls:

- a) Rigid type
 - 1) Reinforced cement concrete.
- b) Flexible type
 - 1) Plastic concrete,
 - 2) Cement bentonite slurry trench, and
 - 3) Earth backfilled slurry trench.

5.2 Diaphragm wall, single line or two lines spaced about 3 to 4 m apart can be provided as per the requirements of water tightness and site conditions. In the latter case, the alluvium contained between the two diaphragm walls is also grouted to achieve better water tightness, if warranted.

6 SELECTION OF TYPE OF DIAPHRAGM WALL

6.1 Selection of type of diaphragm depends upon a number of factors such as:

- a) site conditions;
- b) heterogeneity/perviousness of sub-surface strata;
- c) geological features;
- d) depth of overburden;
- e) anticipated stresses and deformations due to embankment construction and reservoir loading conditions;
- f) availability of construction materials; and
- g) techno-economic considerations, etc.

6.1.1 Guidelines given in this standard are only an indication of the preferred type of diaphragm wall. The decision of the design engineer backed by assessment of various factors, past experience and his own judgement are important in selecting a type of diaphragm wall to suit the project requirements.

6.2 Where the foundation strata consists of bouldary deposits, open gravel pockets, zones of talus, slide areas, contacts of formation of different geological age, fractured zones or similar features, it is obvious that voids/openings may be larger and contact with diaphragm wall face may not be continuous. Under such circumstances it is important to see that bending stresses between contact free lengths are taken care of. Any flexible type of diaphragm wall which undergoes large deformations at low stress levels is not suitable. Rigid type diaphragm wall is preferable under such conditions.

6.3 If stratum in the foundation is fairly uniform and pervious, the contact of soil particles with the face of diaphragm wall will be continuous. Application of rigid diaphragm wall in such strata induces higher stress level due to low deformability of the fill material of the diaphragm wall, when pervious strata undergoes strain on load application. This causes interface problems between diaphragm wall faces and surrounding strata which may ultimately result in cracks between soil and face of diaphragm wall leading to probability of hydraulic fracturing. Under such circumstances the ideal material for the diaphragm wall would be one which possesses deformation characteristics compatible with the surrounding soil mass. Flexible type of diaphragm wall is, therefore, preferable in such circumstances.

6.3.1 Depending on stress-strain behaviour of strata and anticipated stress level upon loading, selection should be made on either plastic concrete

or cement bentonite diaphragm wall or earth back-filled slurry trench diaphragm wall.

6.3.2 If sub-surface strata consists of gravelly or bouldary deposits with the interstices filled up with finer soil particles forming a tight matrix of relatively low permeability, plastic concrete diaphragm is preferable to rigid or cement bentonite slurry trench diaphragm.

7 NECESSARY INFORMATION

7.1 Following information is generally necessary for the design and construction of a diaphragm wall for under-seepage control:

- a) Contoured site plan and detailed drawings of embankment dam/barrage/weir with properties of soil in various zones of embankment dam, showing FRL, MWL, LWL, TWL, etc.
- b) Detailed sub-soil investigations along the proposed alignment of diaphragm wall as laid down in IS 1892 : 1979 and IS 6955 : 1973 with particular reference to the bouldary/gravelly formations.
- c) Longitudinal section along the proposed alignment of diaphragm wall marked with ground levels, sub-surface geological features, type of structure(s) at both ends of diaphragm wall and any other pertinent details.
- d) *In-situ* permeability test data.
- e) Stress-strain curves of embankment and overburden soils obtained in triaxial shear test in appropriate test conditions, covering full depth of overburden.
- f) Physical, physico-chemical and engineering properties of the sub-soil and chemical properties of ground water.
- g) Modulus of sub-grade reaction of the sub-soil in horizontal direction.
- h) Stage of construction or details of the construction sequence and construction schedule of the embankment dam/barrage/weir.
- j) Ground water table, its probable fluctuations and subterranean flow conditions, if any.
- k) Size, weight, capacity and working principle of the trenching and concreting machinery intended to be used.
- m) Quality and amount of local materials available economically at, or near, the site of construction for preparing the slurry, plastic concrete, R.C.C.
- n) Complete details of auxiliary construction equipment like lifting cranes, tower cranes,

hauling units, mixing devices, pumping devices, etc, intended to be used.

8 MATERIALS

8.1 Cement

Cement shall be ordinary Portland cement conforming to IS 269 : 1989 or rapid hardening Portland cement conforming to IS 8041 : 1990, blast furnace slag cement conforming to IS 455 : 1989 or Portland pozzolana cement conforming to IS 1489 (Part 1) : 1991 or IS 1489 (Part 2) : 1991.

8.2 Aggregate

The aggregates shall conform to IS 383 : 1970. Well graded coarse aggregate of 20 mm size shall normally be used in reinforced concrete diaphragm walls. For plastic concrete diaphragm walls, a smaller size of aggregate may be used.

8.3 Sand

Well graded sand consisting of 50 percent coarse sand shall be used.

8.4 Water

Clean water free from deleterious impurities, as specified in IS 456 : 1978, shall be used in preparing the concrete mix and for preparation of bentonite slurry. Water shall be free from salinity when used with bentonite.

8.5 Admixtures

If required, chemical admixtures in concrete shall be used as specified in IS 456 : 1978.

8.6 Reinforcement

Mild steel bars conforming to IS 432 (Part 1) : 1982 and cold worked bars conforming to IS 1786 : 1985 shall be used.

8.7 Bentonite

Bentonite used shall conform to IS 12584 : 1989.

8.8 Clay

Clay shall conform to IS 1498 : 1970. Clay is generally used to reduce consumption of bentonite. Clay is also required to be used in earth-backfilled slurry trench cut-off walls.

9 DESIGN CONSIDERATIONS

9.1 General

9.1.1 Utmost consideration shall be given, while designing the diaphragm wall, so as to achieve:

- a) perfect embedment at both the ends, to avoid/minimize possibilities of cracking both within and surrounding the diaphragm wall, and

- b) imperviousness or water tightness. Water tightness is mainly adjudged with the value of permeability (K). If the water is sufficiently expensive to justify a high cost of diaphragm wall then the diaphragm may be constructed to achieve a permeability of 1, 2 or 3 lugeons. If the seepage loss is little or of negligible value, then water tightness up to a permeability of 3 lugeons is preferred to prevent piping of the foundation material. Permeability of 3 to 5 may be permitted in exceptional cases where an inordinately high expenditure is anticipated for achieving of reduction of up to 3 lugeons.

These considerations will take care of the intended functions of the diaphragm wall as an under-seepage control measure.

9.1.2 Diaphragm wall is a component of the structure for which dimensions are predetermined. The most important consideration in the design of a diaphragm wall is to form an impervious wall having flexibility to avoid cracking. The potential for cracking induced by stresses in the diaphragm wall as construction progresses and during first filling of the reservoir shall be considered, particularly for rigid type of wall. Stresses are usually not a problem unless they cause excessive cracking which would negate the purpose of the wall. For flexible type of wall, its imperviousness, deformability compatible with surrounding soil and required unconfined compressive strength are the governing factors.

9.2 Location

9.2.1 The best location for a diaphragm wall within a structure is where the loads are reasonably balanced on both sides of the wall. In case of new earth/rockfill dam, the axis at top of the dam will be an ideal location due to more or less symmetrical design. At this location bending moments about the horizontal axis will be minimum and the horizontal deformations will also be minimum. However, for any location of the diaphragm wall, the junction of diaphragm wall with its surroundings has to be properly taken care of to avoid piping problems through interfaces (*see 9.4.2*).

9.2.2 An alternative location of the diaphragm wall is at the upstream toe of earth dam which minimizes the possibility of compressive failure of the wall due to negative skin friction developed by settlement of foundation strata under the weight of embankment. For this location, however, construction of embankment shall be carried out first followed by construction of the diaphragm wall. Top

of diaphragm wall should be connected with a horizontal impervious blanket which shall be linked with the upstream toe of impervious core.

9.2.3 Location of diaphragm wall is often influenced by site conditions. Location shall, therefore, be decided after careful study of site requirements and localised features.

9.2.4 Locations of the diaphragm wall required for the barrage/weir are fixed from design point of view. These are generally provided at upstream and downstream ends of the floor of the barrage/weir. However, the junction of the diaphragm wall, other than rigid type of diaphragm wall, with the concrete/masonry work has to be properly thought of. Generally the barrage/weir floor is constructed of rigid concrete or masonry with concrete cover. The junction of rigid concrete in the floor with the flexible type of diaphragm wall may not be monolithic and hence in such cases seepage can take place through such joints. It is, therefore, preferable to use rigid concrete diaphragm walls to act as cut-offs for barrage/weir structures.

9.3 Choice of Panel Dimensions

9.3.1 Choice of panel dimension depends on various considerations as below:

- a) *Stability*: In general, short panels are stabler than long panels. However, in soft ground short panels are preferred.
- b) *Concreting*: It shall be possible to place the total volume of the concrete for a panel before setting, or significant stiffening, takes place.
- c) *Excavation Equipment*: It shall be ensured that the panel length shall not be shorter than a single bite of the grab.
- d) *Number of Stop-End Tubes*: Placing and extraction of stop-end tubes is costly.
- e) *Reinforcement*: Size and weight of each reinforcement cage must allow easy handling with available equipment at site.

9.3.2 Each joint is a potential source of seepage. It is desirable, therefore, that the number of panel joints shall be kept to the minimum. An appropriate size of the panel shall be selected on the basis of considerations (a) and (b) above. This size shall then be analyzed for consideration (c) and then the remaining considerations taken care of. Panel volume shall be of reasonable size and moderate in length because of the possible shrinkage of panels. Final checks shall be made on the closing panels.

9.3.3 A practical maximum panel length shall be of the order of 10 m wherein concrete can effectively be placed using two tremie pipes in a panel of 600

mm thickness. Minimum panel length shall be limited to 5 m, or single bite of the grab, with one tremie pipe for ensuring due concreting operations.

9.3.3.1 In case of flexible type of diaphragm wall possibility of seepage through joints gets reduced to a greater extent. In such cases panel length of 3 m to 5 m will be convenient. Problems of instability of sides of trench and handling/circulating the slurry will be minimum with these panel lengths.

9.3.4 In general diaphragm walls are of thickness 500 mm, 600 mm, 750 mm and 900 mm. These widths have been found to be satisfactory up to a depth of 50 m. For greater depths thicker walls (1 000 mm to 1 500 mm) may be used ensuring trench stability and resistance to hydro-fracturing.

9.3.5 Depth of the wall will depend on the depth to an impervious layer, or a layer that satisfies the designer's criteria for allowable seepage losses, and flow pattern.

9.4 Joints, Junctions and Sub-surface Grouting

9.4.1 Joints

9.4.1.1 In case of rigid diaphragm wall, joints between the primary and secondary panels are generally achieved by use of form tubes (stop-end tubes). The concrete faces of the primary panels are roughened using special tools prior to concreting the secondary panels.

9.4.1.2 In case of a rigid type single diaphragm wall, the joints between the successive panels are generally grouted in accordance with IS 4999 : 1991 on upstream face, by method of grouting through tubes with sleeves (*tube-e-manchette* grouting method) which is usually adopted for grouting of alluvium. In doing so, joints between the panels are sealed from the upstream side.

9.4.1.3 In case of two lines of rigid type of diaphragm walls, alluvium in between the two walls shall be grouted, if warranted, by the method described in 9.4.1.2 to achieve better reduction in permeability.

9.4.1.4 Flexible type of diaphragm walls can be constructed without any joints as a continuous construction process. However, faces of primary panels are required to be roughened by grab teeth or by special tools, prior to filling the secondary panels to get a better joint.

9.4.1.5 Interface between the bottom end of diaphragm wall and bedrock/impervious strata shall be grouted to achieve water tightness. This will be achieved while grouting the strata below the bottom of the diaphragm wall.

9.4.2 Junctions

9.4.2.1 Junctions of the diaphragm wall will generally be at the following locations:

- Junction of the bottom end of the diaphragm wall with sub-surface strata.
- Junction of the diaphragm wall at both ends with hill abutment/any structure.
- Junction of the top end of the diaphragm wall within the structure.

9.4.2.2 Bottom end of the diaphragm wall shall be keyed to a depth of 600 to 1 000 mm into impervious soil or rock.

9.4.2.3 At the junctions of rigid and flexible types of wall with hill abutments, or any structure, it is necessary to provide adequate creep length to get safe exit gradients. Diaphragm walls shall, therefore, extend into the abutment or any structure to a minimum length which provides safe seepage gradient not exceeding 4 horizontal to 1 vertical. Panel length adopted for construction of diaphragm walls should, however, be adjusted according to the required extension.

9.4.2.4 Top end of diaphragm wall, whether of rigid or flexible type, will remain either below the ground level or will extend above the ground level into an impervious core. Extent of the embedment of diaphragm wall into the core can be better decided with the help of finite element analysis. Without such study, extension of the diaphragm wall into core will be based on the designer's judgement. Excessive internal hydraulic gradient can be avoided by proper selective treatment around the diaphragm wall in core, type of impervious soil, width of impervious core, downstream drainage arrangement, etc. In absence of finite element study it is apprehended that the diaphragm wall can damage the impervious core as the foundation settles under the weight of the embankment and sub-

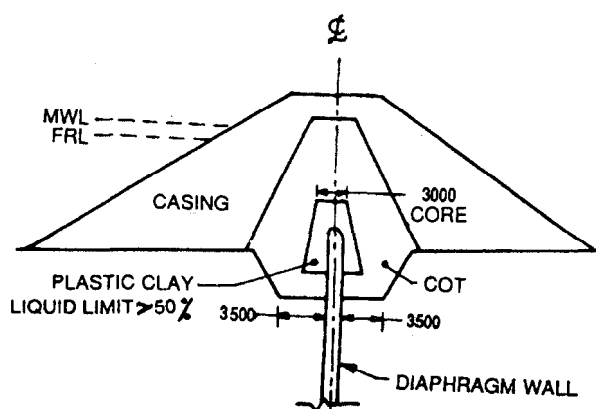
sequent loadings. During earthquake, top portion of the diaphragm wall will be more vulnerable to damage. Extension of rigid type of diaphragm wall is not desirable from such considerations. Hence, precautions shall be taken to prevent possible damages along the extended length of diaphragm wall into the core. Alternatively, it would be preferable to provide a partial cut-off-trench below impervious core to a depth required by permissible internal hydraulic gradient considerations, and to keep the top of diaphragm within the impervious backfill of the partial cut-off-trench up to ground level, or below. This also reduces the possibility of damage to the diaphragm wall near the ground during earthquake.

9.4.2.5 Top end of the diaphragm wall shall be rounded to avoid stress concentration at the corners.

9.4.2.6 Whether the diaphragm wall is extended into the impervious core, or is kept below the ground level within partial cut-off-trench, a cover of plastic clay shall be provided around the diaphragm wall to safeguard against any possible damage to the wall or to the impervious core. This cover shall consist of clayey soil of relatively high plasticity with its liquid limit more than 50. This clay cover will act as a deformable 'cushion' and will reduce stress concentration. It will also improve the interface contacts. Dimensions of cover above top of diaphragm wall shall, however, be decided on available literature/experience/model study, etc. Such treatment will be required for both the rigid and flexible type of diaphragm walls.

9.4.2.7 Junction of flexible type of diaphragm walls with rigid structures shall be avoided.

9.4.2.8 Typical junction details of rigid/flexible diaphragm wall with impervious core are shown in Fig. 1.



All dimensions in millimetres.

FIG. 1 JUNCTION DETAILS OF DIAPHRAGM WALL WITH IMPERVIOUS CORE

9.4.2.9 Typical junction details for diaphragm wall located upstream of the toe of earth dam are shown in Fig. 2.

9.4.3 Sub-surface Grouting

9.4.3.1 The bed rock below the rigid and flexible type of diaphragm wall is generally grouted to a required depth by providing pipes inside the panels while casting, to reduce the permeability of bed rock. Drilling the holes directly into the diaphragm wall after its casting shall be avoided as it is practically impossible to keep the holes within the wall. Also in doing so there is a risk of disturbing the panel material and thereby damaging the wall during drilling.

9.5 Loading on Diaphragm Wall

9.5.1 A diaphragm wall is a structural component embedded in soil, analysis shall, therefore, involve soil-structure interaction. A diaphragm wall, whether rigid or flexible, is subjected to various loading conditions such as construction of structure for which diaphragm wall is provided, first reservoir filling and subsequent operating conditions. Hence, loading conditions on a diaphragm wall are complex and need careful consideration. Loading considerations also depend upon method of analysis selected for assessing the behaviour of diaphragm wall as well as its location. End of construction condition may not be of much importance if diaphragm wall is so located that loading on either side is reasonably symmetrical. However, the diaphragm wall located near the upstream toe of the dam/barrage/weir and constructed prior to construction of dam/barrage/weir, may undergo deformations in horizontal and vertical directions. In such cases loads due to stresses in horizontal direction, on account of post construction of dam/barrage/weir, are of importance.

9.5.2 In subsequent stages, the maximum loads on diaphragm wall are expected when reservoir is fully charged to full reservoir level (FRL) and the structure as well as the foundation soil on the upstream of the diaphragm wall are in completely saturated

state with development of steady state of seepage within the embankment. Loads acting on the upstream face of the diaphragm wall may be considered as follows:

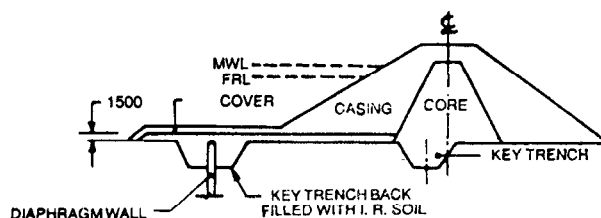
- Effective lateral pressure exerted by the foundation soil over the depth of diaphragm wall between ground level and its bottom.
- Effective lateral stress component contributed by the structural load over the entire depth of diaphragm wall.
- Hydrostatic pressure acting perpendicular to the face of the diaphragm wall considering head of water corresponding to FRL.

9.5.3 If an analysis is carried out by finite element or similar type of method, it is possible to incorporate all the intermediate loading stages involving effect of sequential construction. Otherwise wall loadings mentioned in 9.5.1 and 9.5.2 may be adopted, as the case may be.

9.6 End Conditions

9.6.1 Flexible types of diaphragm walls are designed to undergo deformations under loading. Hence their structural behaviour in bending is not of much importance. Only rigid types of diaphragm walls need structural analysis to take care of bending stresses without developing appreciable deformation which may result in cracking.

9.6.2 Structural behaviour of rigid type of diaphragm wall is different from any isolated structural member, as it involves soil structure interaction. Till date, it has remained a matter of debate about end conditions to be adopted for structural design of the diaphragm wall. Considering the embedment depth in impervious stratum negligible at bottom as compared to the total depth of diaphragm wall and upper end embedded in soil, the end conditions can be considered as hinged at both the ends. This shall be considered as a guide only. Depending upon his own experience and/or with the help of finite element study, the designer may select suitable end conditions.



All dimensions in millimetres.

FIG. 2 JUNCTION DETAILS OF DIAPHRAGM WALL LOCATED IN THE UPSTREAM OF THE TOE OF EARTH DAM

10 GUIDELINES FOR STRUCTURAL DESIGN

10.1 Rigid Type of Diaphragm Wall

10.1.1 Structural Analysis

Rigid type of diaphragm wall is to be analyzed either by the method of beam on elastic foundation or by finite element method (FEM).

10.1.1.1 Method of beam on elastic foundation

The diaphragm wall shall be considered in plane strain state and as such, unit length of diaphragm wall with entire depth as the span is to be considered as a beam resting on elastic soil media on the downstream face. Loads are to be considered acting upon the upstream face of the diaphragm wall. Analysis is to be carried out using appropriate equations for bending moments and shear stresses for a beam resting on elastic foundation with assumed end conditions.

10.1.1.2 Finite element method (FEM)

Finite element analysis takes into account soil structure interaction. Finite element analysis shall be carried out either as gravity turn-on analysis or sequential construction analysis. Sequential construction analysis is, however, preferred as it takes into account the elastic modulus of soil changing with different stress levels during construction. The finite element analysis shall incorporate interface elements along the contact boundary of the diaphragm wall and surrounding soil mass. Elimination of interface elements results in faulty stress and displacement computations, due to stress transfer through common nodes.

10.1.1.3 Finite element method takes considerable time for analysis as well as testing. This affects the project schedule. On the other hand, a simplified method like beam on elastic foundation, in which sequential construction cannot be accounted for is not a refined method like FEM. As a compromise it is recommended to analyze the diaphragm wall by the method of beam on elastic foundation and to carry out finite element analysis simultaneously as construction goes on. After completion of the diaphragm wall, performance shall be evaluated by comparison of instrumentation observations of stresses and displacements with those obtained by finite element analysis.

10.1.2 Concrete Mix

10.1.2.1 Concrete for rigid type of diaphragm wall shall be of M20 grade (see IS 456 : 1978). Where required, concrete mix design for rigid type of diaphragm wall shall be done to obtain specified compressive strength.

10.1.2.2 The water-cement ratio of concrete used shall ensure easy flow through tremie pipe used in concreting. However, the water-cement ratio shall not be greater than 0.6. The concrete mix shall have the required slump. 10 percent extra cement shall be added for under water work.

10.1.3 Design of Reinforcement

10.1.3.1 Steel reinforcement is required to be provided in case of rigid type of diaphragm wall to resist bending moments and shear stresses computed as mentioned in 10.1. The design of reinforcement that is, size and spacing of bars, minimum steel requirements etc, shall be carried out in accordance with IS 456 : 1978.

10.1.3.2 An allowance of 10 percent to 20 percent shall be made for a reduction in bond strength due to the probability of lodging of sand-laden gelled slurry in case of deformed bars.

10.1.3.3 Reinforcement cage shall be designed for sufficient strength to resist handling stresses. Cages shall be able to resist uplift pressures during tremie concreting and as such adequate tie bars shall be provided. It is imperative that the tremie pipe passes between faces of the reinforcement cage freely, so that, it is possible to lift and lower the tremie pipe frequently during concreting.

10.1.3.4 Spacers shall be provided to the outside horizontal main reinforcement bars at frequent intervals along the height of cage, to place the cage in centre of the trench, and for easy lowering of the cage.

10.1.3.5 Reinforcement in the diaphragm wall is placed in the form of cages. A single cage can be prepared if it is practicable to lift the cage and to place it in position without distorting the cage. Where it is not feasible to lower the cage in one section of full height, due to the limitations of the equipment, site considerations etc, the cage shall be fabricated in sections. After lowering the first cage section and keeping it just projected above the guide walls the following cage section shall be welded keeping it suspended above the lowered one.

10.1.3.6 Connections between the reinforcement bars and other steel sections shall be welded with design considerations as well as to withstand handling stresses.

10.1.3.7 To ensure easy flow of concrete, minimum spacing between reinforcing bars shall not be less than 100 mm.

10.2 Flexible Types of Diaphragm Wall

10.2.1 Structural Analysis

10.2.1.1 Plastic concrete diaphragm wall or cement bentonite slurry trench diaphragm wall or

earth backfilled slurry trench diaphragm wall are relatively flexible and capable to deform under stresses in the surrounding soil. Hence, development of stresses is not a design problem. These diaphragm walls shall, therefore, be designed to undergo deformations compatible with those in the surrounding soil without development of cracks.

10.2.1.2 As a whole, the diaphragm wall will deform due to the following reasons:

- a) Due to the settlement of the structure, that is, dam/barrage/weir under its own weight;
- b) Due to the settlement and deflection of the foundation beds under the weight of the structure;
- c) Due to the horizontal deflections during first filling of the reservoir; and
- d) Subsequently to the alternating loads related to the use of the reservoir water.

Diaphragm walls shall, therefore, be constructed of materials which possess characteristics to withstand/adjust with such deformations imposed without cracking. Ideal materials are those which give the deformation characteristics similar to those of the surrounding soil. In case of homogeneous sub-surface soil, or when the variation of Young's Modulus of overburden soil with depth is of low magnitude, materials having Young's Modulus 4 to 5 times greater than the over-burden soil are adequate.

10.2.1.3 Evaluation of the behaviour of the diaphragm wall shall be made either by elastoplastic method or by the finite element method.

10.2.1.4 The governing criteria is that the diaphragm wall shall be impervious.

10.2.2 Concrete Mix

10.2.2.1 For plastic concrete diaphragm wall the water cement ratio shall not be greater than 0.5.

10.2.2.2 Mix design for flexible types of diaphragm wall needs careful consideration. Modulus of elasticity of surrounding soil shall be obtained by conducting adequate number of triaxial shear tests in conditions appropriate for loading. Mix shall have modulus of elasticity 4 or 5 times greater than that of the surrounding soil. The modulus of elasticity of the mix shall be obtained by conducting triaxial

shear tests in the same condition under which surrounding soil specimens are tested.

10.2.2.3 Mix shall have adequate unconfined compressive strength to take the weight of the structure, to resist the soil stresses at depth and to resist erosion.

10.2.2.4 In addition to requirements stated above, the mix shall also possess other properties, that is, durability, erodibility and workability.

10.2.2.5 Mix for plastic concrete/cement-bentonite shall have permeability of 10^{-6} to 10^{-7} cm/s.

10.3 Earth Backfilled Slurry Trench Diaphragm Wall

10.3.1 In this type of diaphragm wall the backfill is generally made by mixing the *in-situ* materials obtained with slurry during excavation of the trench and earth material from additional sources, if required. As cement is not used, it will not be possible to obtain compressive strength. Hence imperviousness is the governing criteria.

10.3.2 Material to be used from additional sources shall be impervious clay. Moreover, materials excavated from the trench with slurry shall be the finer material obtained after screening to remove the coarser fractions. Thus, as such, mix design is not required for this type of diaphragm wall.

11 SLURRY TRENCH AND REQUIREMENTS

11.1 Requirements of Slurry

11.1.1 Bentonite slurries, used for supporting excavation shall fulfil the following requirements :

- a) Support the excavation by exerting hydrostatic pressure on its walls.
- b) Remain in the excavation area and not flow into the soil.
- c) Suspend detritus to avoid sludgy layer building up at the excavation base.
- d) Clean displacement by concrete.
- e) Screening or hydrocycloning to remove detritus and enable recycling.
- f) Easy pumping.

To achieve these requirements the slurry shall have properties as given in Table 1. However, they shall be confirmed to suit the site conditions.

Table 1 Requirements of Slurry
(Clause 11.1.1)

Property	Method of Test	Permissible Value at 20°C
Density	Mud balance or hydrometer	1.04 to 1.25 g/ml
pH value	pH indicator paper strips	9.5 to 12
Viscosity	Marsh cone method	28 to 42 seconds
Plastic	Fann Viscometer	< 20 cP
Viscosity	Shearometer	1.4 to 10 N/m ²
10-minutes gel strength	or vane shear apparatus	(14 to 100 dyne/cm ²)
Sand content	75-microns sieve	1% to 25%

11.1.2 Sodium bentonite powder shall be thoroughly mixed with potable water to form a fully dispersed lump-free homogeneous slurry. Suitable slurry tanks shall be used for this operation. Use of a slurry pump with special nozzle (see Fig. 3) is suggested for preparing bentonite slurry. Use of paddle stirrer or other mechanical devices such as colloidal grout mixer (see Fig. 4), may also be made for proper mixing of slurry. Temperature of water and the slurry used shall not be less than 5°C.

11.1.3 Relationship between concentration C of bentonite slurry expressed as percentage by mass and the density r_s is given below:

$$r_s = 1.0 + 0.006 C$$

NOTE — This relation is valid for Indian bentonites and represents an average sample. There may be some variation of bentonites. Laboratory calibration may be prepared for the bentonite samples actually used.

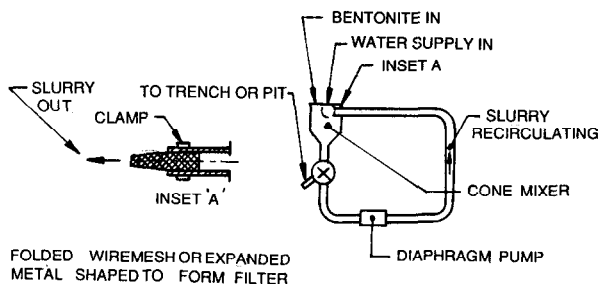


FIG. 3 SPECIAL NOZZLE AND SLURRY MIXING PLANT

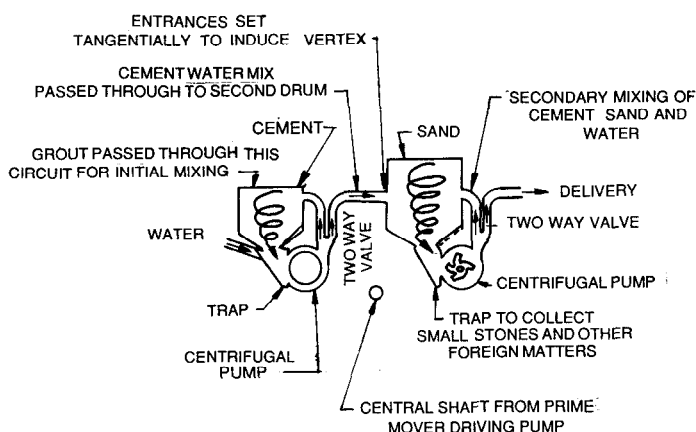


FIG. 4 FLOW DIAGRAM OF CONCRETE DOUBLE DRUM COLLOIDAL GROUT MIXER

11.1.4 Table 2 gives guidance on the proportion of bentonite suspension to be used for different sub-soils based on Marsh Cone viscosity.

11.1.5 Tests to determine density, viscosity, shear

strength and pH value shall be carried out until a consistent working pattern is established, taking into account the mixing process, blending of freshly mixed bentonite slurry with previously used slurry

Table 2 Proportions of Bentonite Suspension for Different Sub-soils
(Clause 11.1.4)

Type of Sub-soil	Proportion of Slurry, Min	Viscosity by Marsh Cone, s
Sand coarse or mixed with pebbles	1:10	40-42
Sand medium	1:12	35-40
Silty sand or loam	1:16	33-35
Soft rocks	1:20	26-32

and any process which is to be employed to remove impurities from previously used bentonite slurry.

11.1.6 When results show consistent behaviour, the tests for shear strength and pH value may be discontinued and tests required to determine density and viscosity be carried out.

11.1.7 Frequency of testing shall be on panel to panel basis where bentonite slurry becomes heavily contaminated with fine sand during its first use, and on a daily basis where slight contamination is expected. In cases where a mechanical process is employed to remove contaminating solids from the slurry, frequency of slurry testing shall depend on equipment employed.

11.1.8 Prior to placing of concrete in any panel a bentonite slurry sample shall be taken (about 0.2 m from the trench bottom) and the same shall be tested for density. The sampling shall be done carefully by an appropriate method. The density thus determined shall not be greater than 1.25 g/ml to ensure satisfactory placing of concrete. If the slurry is found to have heavier density, the same shall be thinned till the required density is achieved.

11.1.9 Suitable slurry pumps, submersible pumps or air lift pumps shall be used for replacing the contaminated slurry at the bottom of trench by fresh bentonite slurry.

11.2 Stability of Slurry Filled Trenches

11.2.1 Mixing of bentonite in water forms slurry, which is used during excavation of the trench for diaphragm wall. Water from the slurry bleeds into the sides of the trench and leaves behind a thin densely packed collection of colloidal particles commonly referred to as a 'filter cake'. Hydrostatic force of the slurry, acting in combination with filter cake, provides stability to the sides of trench.

11.2.2 Bentonite slurry filled in the trench imparts stability mainly by applying hydrostatic pressure on the wall, against the impermeable thin film formed along the wall. Secondly, the slurry filled in the trench provides passive resistance against failure of the trench, and thirdly, the shearing resistance of the slurry saturated zone and the plastering

effects of the filter cake also contribute towards trench stability. Hydrostatic pressure alone represents 65 percent to 80 percent of the total stabilizing forces. If the density of slurry used can provide a factor of safety of one, due to hydrostatic pressure, then the factor of safety of the actual trench shall be between 1.25 to 1.50. Therefore, taking only hydrostatic pressure and considering $F=1$, the density of slurry may be calculated as indicated by the following formula. This formula shall be used as a guide only.

A For clayey soils :

$$F = \frac{N_c C_u}{H(r - r_s)}$$

where

H = depth of the trench,

C_u = undrained shear strength of clayey soils,

r = natural density of saturated soil,

r_s = density of the slurry needed for the trench, and

N_c = bearing capacity factor which varies from 4 at the ground surface to 8 for deeper depths, depending upon D/B and L/B ratio of the trench. This factor accounts for arching action in horizontal as well as vertical directions (see Fig. 5).

B For sandy soils :

$$r_s = r_w + A (K_a r')$$

where

$$A = \frac{1 - e^{-2n k_a \tan \phi}}{2n k_a \tan \phi}$$

$K_a = \tan^2(45^\circ - \phi/2)$,

ϕ = angle of internal friction,

r' = effective unit weight of the sandy soil,
= $r_{sat} - r_w$,

r_{sat} = saturated unit weight of soil, and

r_w = unit weight of water.

The value of A depends upon depth/length ratio of the trench ($=n$) (see Fig. 6). As a general rule, level of bentonite slurry in the trench shall be adequately higher than the water level.

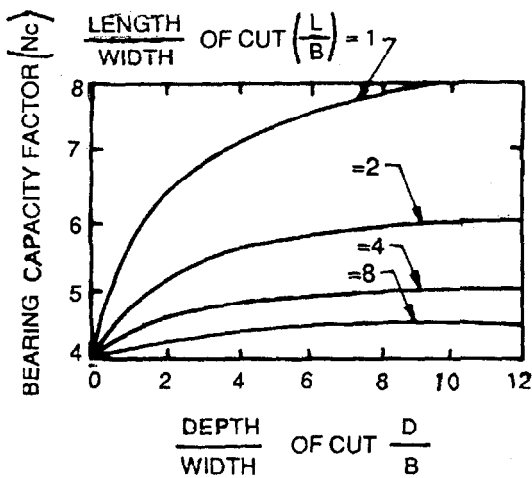


FIG. 5 STABILITY FACTOR N_c FOR RECTANGULAR CUTS IN CLAY

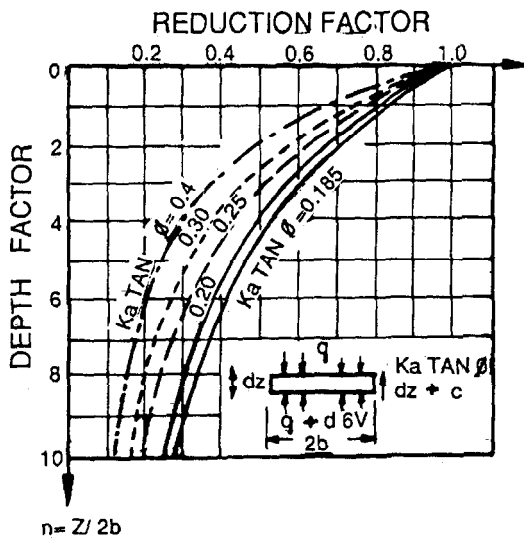


FIG. 6 REDUCTION FACTOR A FOR EARTH PRESSURE IN TRENCH OF LENGTH $2B$ ($cd=0$)

11.3 Additives

11.3.1 Where saline or chemically contaminated ground water is present, special additives as listed below, have to be used to render bentonite slurry fit for use. These additives shall be used in very small amounts of 0.1 percent to 0.5 percent by mass of the slurry.

- Ferrochrome lignosulphonate in combination with soda ash or bichromate of soda, shall be used for effective bentonite hydration, if hardness of sea water exceeds 200 ppm.
- Sodium carboxymethyl cellulose (SCMC) is another additive sometimes used. It protects slurry against effects of electrolytes, accelerates filter cake forma-

tion, and reduces fluid loss by increasing the viscosity of slurry.

- Cement contamination is counteracted by phosphates. Calcium gets removed and clay solids get dispersed. Phosphates decrease pH value thereby lowering viscosity and yield value of slurry.
- Carboxymethyl cellulose, gums or presheared asbestos may be used to increase viscosity and reduce filter loss.
- To remove fine silty solids and clay solids from the slurry, flocculants such as vinyl acetate maleic anhydride co-polymer or polyacrylamides shall be used. Guar gum can flocculate clays, carbonates, etc.
- Pregelatinized starch can be employed as a fluid loss control. It can also be used as a protective colloid against the effect of electrolytes.

12 EQUIPMENT AND ACCESSORIES

12.1 Trenching Equipment

Depending upon the type of soil encountered at the site and the depth, length and thickness of the diaphragm wall to be constructed, suitable trenching equipment shall be chosen. General trenching equipment shall include rotary boring rigs, percussion boring rigs, trenching bucket type shovels, mechanical grabs, hydraulic grabs, grabs with kelly bars, grabs controlled by suspended wire ropes of a crane, direct mud circulation boring rigs, reverse mud circulation rigs and submersible mortar drills for trenching equipment. For gravelly soils, boulder deposits and rock formations, specially designed chiselling equipment shall be considered. If required, methods in combinations of the above may be used.

12.2 Slurry Preparation and Testing Equipment

Tanks of suitable sizes and slurry pumps of suitable capacity shall be used for storage, mixing and circulation of bentonite slurry at the site. A separate water pump shall be used for water supply to slurry tank. Equipment for sampling the slurry from deep trenches and testing its concentrations, viscosity, pH value and hardness of ground water, in which the bentonite slurry and concrete are prepared, shall also be used. Testing of slurry after contamination with soil or cement indicates the need of disposal or reuse as the case may be. Vibrating screens, hydrocyclones and centrifuges for cleaning the bentonite slurry for reuse shall be employed.

12.3 Concreting Equipment

Concrete mixers, tremie pipes of suitable length and size and concrete pouring devices (manual or

mechanical) shall be used according to the need of the work. Lifting arrangement for tremie pipes shall be capable of doing the work with desired speed.

12.4 Lifting Devices

Cranes of suitable capacity and boom length, derrick or any other suitable auxiliary rig shall be used for lowering the reinforcement cage in trench. If the weight of the reinforcement cage and height are small, this work can also be done by winch and pulley arrangement provided on the diaphragm walling rig. Cranes or rigs with winches of adequate capacity may be used for operating the trenching grabs as necessary.

12.5 Grouting Equipment

Panel joints and bed rock below the concrete diaphragm wall have to be grouted. Necessary equipment to perform this job efficiently shall be used.

12.6 General Guidelines

Selection of trenching shall be made to suit the soil conditions. Vibrations produced during construction shall not have any damaging effect or cause any sort of instability to existing structures. Consideration shall be given to selection of equipment which are required to work on a site with restricted space or head room.

13 STAGES OF CONSTRUCTION

13.1 Pre-trench and Guide Walls

A trench is excavated in advance for about 1 m to 2 m depth. Reinforced concrete guide walls of 100 mm to 250 mm thickness are constructed on both the faces of pre-trench (see Fig. 7), they act as guides to trenching equipment during excavations. They also provide support over the trench area subjected to heavy construction surcharge pressures. Besides, guide walls define the planned path of excavation.

13.2 In soft ground or fill, guide walls have to be taken deeper. When ground water table is close to surface, guide walls higher than the surface level shall be constructed to maintain additional slurry head.

13.3 Clearance between finished diaphragm wall and guide wall shall be 50 mm minimum, for straight panels. Clearance shall be suitably increased when the panels are curved.

13.4 Finished faces of the guide walls towards the trench shall be vertical.

13.5 Guide walls after construction shall be suitably propped to maintain specified tolerance.

13.6 Mesh or cage reinforcement shall be used in guide walls.

13.7 Level of slurry in the trench shall be maintained at least up to bottom level of guide walls. However, the level shall be increased if ground water table is higher.

13.8 For heavy trenching machinery, guide walls shall be constructed with suitable ground slab (on both sides of the walls).

13.9 Guide walls get support from adjoining panels and, therefore, their construction shall be done continuously.

13.10 Cast In-Situ Diaphragm Wall

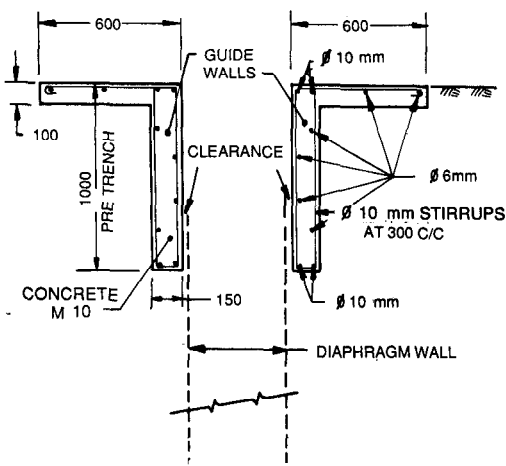
13.10.1 Rigid type of diaphragm wall, or plastic concrete diaphragm wall, shall be constructed by resorting to either successive panel method or alternate panel method. For cement bentonite slurry trench diaphragm wall, or earth backfilled slurry trench diaphragm wall, alternate panel method of construction is suitable in view of the time that is required to achieve hardness of the mix put in the trench.

13.10.2 Successive Panels Method

In this method, a panel shall be cast in continuation of the previously completed panel. Only one form tube is needed for creating a joint with the preceding panel. However, with larger width and greater depth of diaphragm wall, it may not be possible to use form tubes due to handling, lowering and extraction difficulties. In such cases, special tools, like semi-circular chisels, are used to form a joint with the preceding panel. Thus, use of form tubes is eliminated. Rapid construction is possible with this method. However, there is a probability of insufficient hardening of the concrete in the preceding panel with the subsequent possibility of damage.

NOTE — Form tubes of 1 m diameter and 30 m length have been successfully used.

13.10.2.1 Excavation of successive trench panels (see Fig. 8A) shall be done with the help of suitable



All dimensions in millimetres.

FIG. 7 PRE-TRENCH AND GUIDE WALLS

machinery. Trench panels shall be kept filled with bentonite slurry of suitable consistency and viscosity during the excavation period.

13.10.2.2 A stop end tube with a smooth surface or a structural section shall be inserted in the trench at the end of the successive panel to support concrete and to form a suitable joint with the next panel.

13.10.2.3 The reinforcement cage shall then be lowered into the trench panel and shall be suitably supported. Concrete cover for the reinforcement shall be maintained by use of spacers.

13.10.2.4 Before placing concrete in the panel, the trench shall be properly flushed to clean the bottom and to remove thicker suspension from the bottom and lower levels. Density of the slurry shall be checked by taking a sample about 0.2 m from the bottom. If the density of the sample is found to exceed 1.25 g/ml, flushing shall be carried out with fresh bentonite slurry. Concreting in the trench panel shall be done through one or more tremie pipes with suitable funnels (see Fig. 8B).

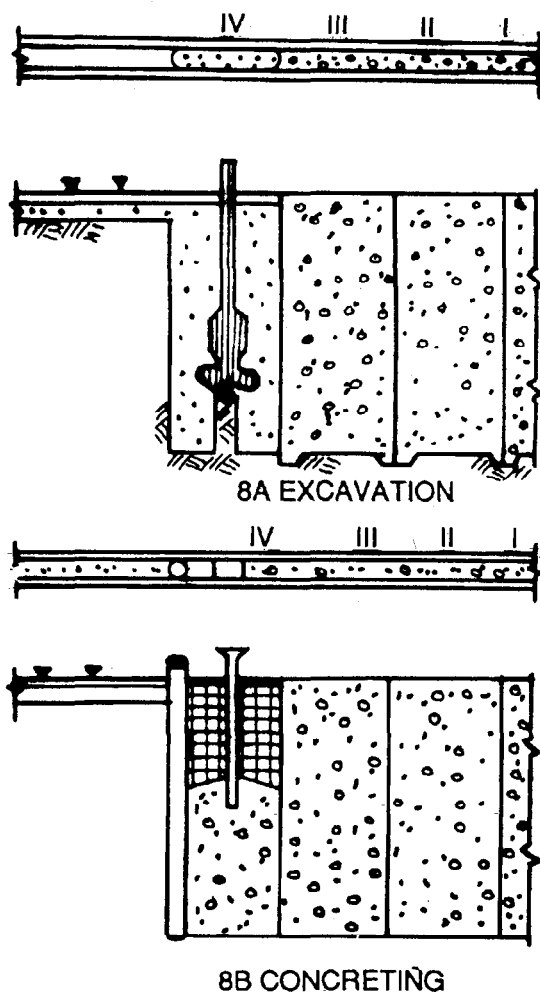


FIG. 8 SUCCESSIVE PANEL CONSTRUCTION
(EXCAVATION AND CONCRETING)

13.10.2.5 Stop end tubes shall be taken out gradually after initial set of concrete. This has to be done carefully with the help of a suitable crane, or any other lifting device, so as not to cause any damage or cracks in concrete at the panel end.

13.10.2.6 Suitable joints shall be adopted to form a watertight joint between the panels.

13.10.3 Alternate Panel Method

13.10.3.1 With alternate panel construction using primary and secondary panels, strength developed in concrete will be sufficient before excavating the adjoining panel. Thus, there is no danger of damage to the concreted panel. In this method, primary panels are cast first leaving suitable gaps in between. Secondary panels are then cast in these gaps (see Fig. 9). Two stop end tubes are used at the ends of the primary panels to support concrete and to form a joint with the secondary panels.

13.10.3.2 Due to the space left by stop-end tubes, primary and secondary panels are of different sizes. The end shape of the panels will also be different. However, shape of the primary end panel shall be such as to form a good joint with secondary panels. Other construction techniques are similar to those of the successive panel method.

13.10.4 Direct Circulation Method

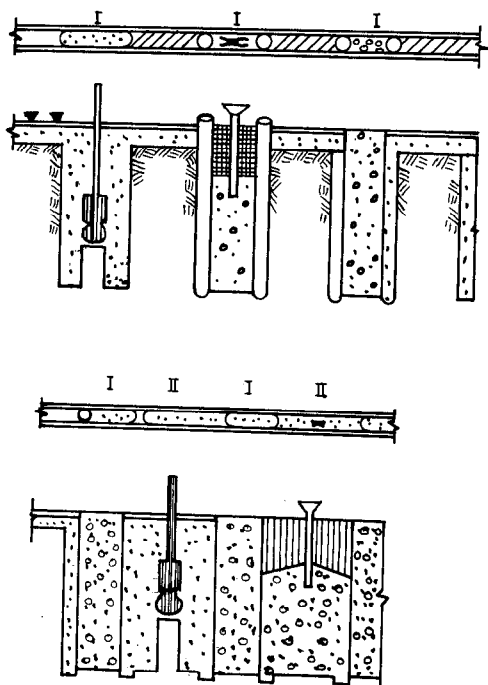
13.10.4.1 This method is used with rotary or percussion type of rigs where drilling fluid (bentonite slurry) is pumped through drilling rods. It can be used for successive panel or alternate panel construction. Stages of construction are shown in Fig. 10. Simple trenching rigs for excavation are used. Circular concreting tremie pipes for backfilling the trench panel have to be used. This method is suitable for shallow depths and for bringing up lighter cuttings.

13.10.4.2 The trench panel is excavated by making overlapping bore holes with bentonite slurry-jet in combination with percussion and to and fro rotary motion of jetting pipe, having a suitable cutter at the tip. A special semi-circular cutter shall be used for providing appropriate shape at each panel end to form a suitable joint.

13.10.4.3 Operations of filling bentonite slurry in the trench and lowering of reinforcement cage into the trench panel are similar to those described under successive panel method.

13.10.5 Reverse Circulation Method

13.10.5.1 The reverse circulation method shall be used to make trench panels as shown in Fig. 11. Forward and backward movement of the rig from one end of the panel to the other, increases the depth of the panel in a zig-zag manner.



I = Primary Panels
 II = Secondary Panels

NOTE — Due to space left by stop end tubes primary and secondary panels are of different sizes and panel end shape.

FIG. 9 CONSTRUCTION OF PANELS

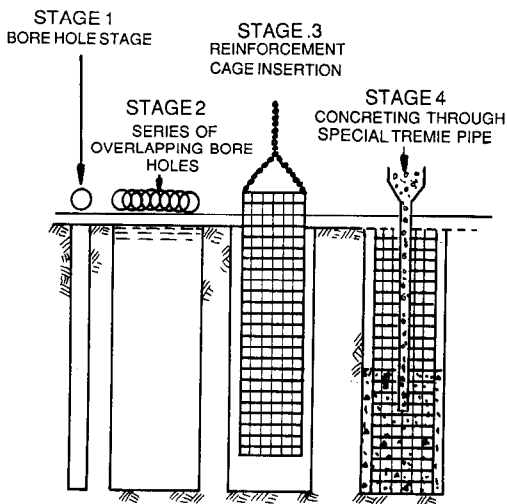


FIG. 10 STAGES OF DIAPHRAGM WALL CONSTRUCTION BY DIRECT CIRCULATION METHOD

13.10.5.2 High capacity pumps shall be used to suck the loosened soil in the slurry filled trench. Separators, or sedimentation tanks, shall be used to retain the soil cuttings and to pass the slurry for circulation and reuse.

13.10.5.3 This method is suitable for greater depths and to bring up heavier cuttings.

13.11 Cement Bentonite Slurry Trench Diaphragm Wall

13.11.1 For cement bentonite slurry trench diaphragm wall it is preferable to use reverse circulation method so that bentonite slurry at the bottom of the trench is sucked out along with the chiselled material.

13.11.2 At the end of excavation, the bentonite slurry in the trench may have much higher density than fresh bentonite slurry due to contamination of soil. This bentonite slurry shall therefore have to be replaced by fresh bentonite slurry having predecided concentration.

13.11.3 The contaminated slurry is to be replaced by keeping the volume of fresh slurry flowing into the trench, equal to the volume of contaminated slurry being pumped out from the bottom of the trench. This is possible as contaminated slurry is heavier than fresh slurry.

13.11.4 After complete replacement with fresh bentonite slurry, samples of the bentonite slurry shall be collected from one-third, two-thirds and bottom depths of the panel to check uniformity of the slurry in the trench.

13.11.5 After filling the trench completely with fresh bentonite slurry, a required quantity of cement shall be added. Estimation of quantity of cement required for a panel has to be done considering full depth of wall, that is, up to platform level and actual width of trench formed. For this purpose, slurry from the trench shall be pumped out from the bottom of the trench into a mixing tank having proper arrangements for mixing with cement. In this tank cement shall be added bag by bag till the entire predetermined quantity of cement gets mixed. While adding cement there shall be a continuous flow of cement bentonite slurry from the trench to the mixing tank and from the mixing tank to the trench by gravity, that is, pumped out and pumped in volume to remain equal. This process shall be continued till required quantity of cement is added to the panel. Cement bentonite slurry shall be kept in circulation for a further 15 minutes to ensure uniformity of mix. At the end of circulation, samples shall be collected from one-third, two-thirds bottom depths of panel to check the uniformity.

NOTE — For proper mixing of cement and bentonite slurry it is very essential to have powerful pumps so that complete circulation of cement bentonite slurry can be achieved for the entire depth of trench within 10 minutes.

13.11.6 After mixing of the cement in a particular panel, excavation for the adjacent panels can be carried out after allowing minimum 7 days for cement bentonite slurry to attain sufficient strength. No form tubes are used for joints.

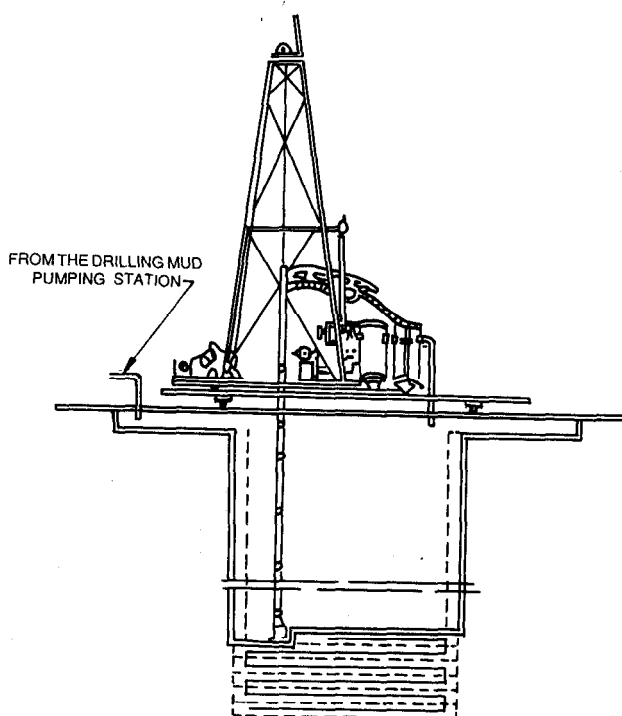


FIG. 11 TRENCH EXCAVATION BY REVERSE CIRCULATION METHOD

13.11.7 Cement bentonite slurry trench diaphragm wall is constructed without any joints between the panels. Hence, while excavating the adjacent panel, over-cutting of the earlier cast panel shall be carried out. In order to ensure proper over-cutting the excavating tool shall be so selected that verticality is maintained.

13.11.8 Bleeding occurring on setting of cement bentonite slurry shall be replaced by fresh cement bentonite slurry, generally after 7 days of casting.

13.12 Earth Backfilled Slurry Trench Diaphragm Wall

13.12.1 An earth-backfill slurry trench diaphragm wall is constructed by excavating a narrow vertical trench having width of 600 mm to 1 500 mm. The process of excavating such narrow trenches is similar to those described in 13.10.2, 13.10.3, 13.10.4 and 13.10.5.

13.12.2 After the trench has been excavated to the final grade, it shall be backfilled with earthen backfill materials. This can be achieved by mixing *in-situ* materials obtained with slurry during excavation of the trench and earth materials from additional sources, as required, to obtain the desired engineering properties. The backfill is placed by

use of tremie, clam-shell or direct pushing into the trench, displacing the slurry.

14 TOLERANCES

14.1 Guide Walls

Finished faces of the guide walls towards the trench shall be vertical. There shall be no ridges or abrupt changes on the face of the guide wall and variations from a straight line or a specified profile shall not exceed 25 mm in 3 m.

14.2 Diaphragm Wall

14.2.1 Verticality

Faces of the wall and ends of the panel shall be vertical within a tolerance of 1:80.

14.2.2 In positioning of reinforcement, longitudinal tolerance of cage head at top of the guide wall, measured along the trench, shall be 75 mm and vertical tolerance at cage head in relation to top of guide wall shall be 50 mm.

15 TESTING

15.1 Testing of materials to be used for the work shall be done in a laboratory to confirm their usability.

15.2 Mix for rigid concrete, plastic concrete and cement bentonite slurry shall be designed in a laboratory and testing shall be done to ascertain various parameters like compressive and tensile strength, permeability, modulus of elasticity, erodibility, pH value, etc, to confirm that the design mix satisfies the design requirements.

15.3 Samples of the mix at the time of placing shall be collected and kept in airtight sealed moulds of specified size till due dates of various tests. Important tests to be performed are as under:

a) *Compressive Strength*

This is the defined stress at which the material will fail. Tests shall be carried out at 7, 28 and 90 days.

b) *Triaxial Compressive Strength*

Triaxial compressive strength tests shall be carried out on filling materials like plastic concrete, cement, bentonite, etc to determine stress-strain characteristics, modulus of elasticity and shear parameters. Tests shall be carried out at 7, 28 and 90 days in consolidated undrained conditions.

c) *Permeability*

Permeability tests on samples shall be conducted after 28 days in a membrane permeameter.

16 INSTRUMENTATION

16.1 Placement of instruments simultaneously with casting of the panel is difficult, as there is a possibility of damaging the instruments, wires, etc, or loosing their sensitivity due to vibrations generated during placement of concrete. It is, therefore, preferable to install the instruments outside the completed diaphragm wall to measure *in-situ* performance.

16.1.1 *Deformation of the Structure*

16.1.1.1 For the purpose of measuring deformation behaviour of the diaphragm, inclinometers at various locations shall be installed close to the diaphragm wall.

16.1.2 *Settlement Gauge*

Settlement gauges may be installed at selected locations to measure vertical displacements. Gauges shall not have an error of more than 1 mm.

16.1.3 *Piezometric Levels*

To judge the efficiency in watertightness of the diaphragm wall, residual discharge collected downstream of the cut-off wall is the essential measurement. This can be determined by a double network of piezometers placed on either side of the wall and well protected by filters. Chemical and physical analysis of the water may be useful to pinpoint its source. Observation frequency shall not be more than 15 days.

16.1.4 *Permeability*

100 mm diameter tubes extending to random depths, shall be placed at the centre of the thickness of the diaphragm panels at randomly chosen locations. After a period of 28 days bore holes may be drilled into the diaphragm below the tubes and permeability tests by pumping in method carried out. The *in-situ* permeability of the diaphragm shall be compared with the specified limit as per requirements.

17 RECORDS

17.1 Following records shall be maintained in a manner approved by the Engineer-in-charge:

- a) Name of project/work;
- b) Panel No. and reference drawing No;
- c) Date of commencement and completion of excavation;
- d) Date of concreting of panel;
- e) Length of panel;
- f) Thickness of panel;
- g) Top of guide wall level;
- h) Depth of guide wall;
- j) Top level of wall as cast, in relation to top of guide wall at the edges and at the centre;
- k) Depth of panel from base of top of guide wall;
- m) Strata encountered;
- n) Volume of panel and volume of concrete used, slump, water-cement ratio;
- p) Cubes taken and their results;
- q) Details of reinforcement (Cage type);
- r) Details of any obstructions/peculiar conditions encountered and time spent and measures taken in overcoming them;
- s) Amount of bleeding observed; and
- t) Type and proportion of any additives used and reasons for use.

ANNEX A

(Clause 2.1)

LIST OF REFERRED INDIAN STANDARDS

<i>IS No.</i>	<i>Title</i>	<i>IS No.</i>	<i>Title</i>
269 : 1989	Specification for 33 grade ordinary Portland cement (<i>fourth revision</i>)	1489 (Part 2) : 1991	Specification for Portland cement: Part 2 Lined clay based (<i>third revision</i>)
383 : 1970	Specification for coarse and fine aggregates from manual sources for concrete (<i>second revision</i>)	1498 : 1970	Classification and identification of soils for general engineering purpose (<i>first revision</i>)
432 (Part 1) : 1982	Specification for mild steel and medium tensile steel bars and hard-drawn steel wire for concrete reinforcement : Part 1 Mild steel and medium tensile steel bars (<i>third revision</i>)	1786 : 1985	Specification for high strength deformed steel bars and wires for concrete reinforcement (<i>third revision</i>)
455 : 1989	Specification for Portland slag cement (<i>fourth revision</i>)	1892 : 1979	Code of practice for sub-surface investigations for foundations (<i>first revision</i>)
456 : 1978	Code of practice for plain and reinforced concrete (<i>third revision</i>)	2116 : 1980	Specification for sand for masonry mortars (<i>first revision</i>)
1139 : 1966	Specification for hot rolled mild steel, medium tensile steel and high yield strength steel deformed bars for concrete reinforcements (<i>revised</i>)	4999 : 1991	Recommendation for grouting of pervious soils (<i>first revision</i>)
1489 (Part 1) : 1991	Specification for Portland cement: Part 1 Fly ash based (<i>third revision</i>)	6955 : 1973	Code of practice for sub-surface exploration for earth and rockfill dam
		8041 : 1990	Specification for rapid hardening Portland cement
		12584 : 1989	Specification for bentonite for grouting in civil engineering works

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Review of Indian Standards

Amendments are issued to standards as the need arises on the basis of comments. Standards are also reviewed periodically; a standard along with amendments is reaffirmed when such review indicates that no changes are needed; if the review indicates that changes are needed, it is taken up for revision. Users of Indian Standards should ascertain that they are in possession of the latest amendments or edition by referring to the latest issue of 'BIS Handbook' and 'Standards Monthly Additions'.

This Indian Standard has been developed from Doc : No. RVD 08 (141).

Amendments Issued Since Publication

Amend No.	Date of Issue	Text Affected

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832 78 91, 832 78 92

Branches : AHMADABAD. BANGALORE. BHOPAL. BHUBANESHWAR.
COIMBATORE. FARIDABAD. GHAZIABAD. GUWAHATI. HYDERABAD.
JAIPUR. KANPUR. LUCKNOW. PATNA. THIRUVANANTHAPURAM.